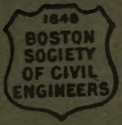
# JOURNAL OF THE
# BOSTON SOCIETY
# OF
# CIVIL ENGINEERS



OCTOBER - 1933

VOLUME XX

NUMBER 8

# BOSTON SOCIETY OF CIVIL ENGINEERS

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# JOURNAL OF THE
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Volume XX OCTOBER, 1933 Number 8

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### OF GENERAL INTEREST

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# COMING MEETINGS

*SOCIETY — WEDNESDAY, NOVEMBER 15*

Joint Meeting with Student Chapters of M. I. T., Tufts, Harvard and Northeastern University

"Radio City" (illustrated), by Walter H. Kilham, Jr., Architect

6.00 P.M. Buffet Supper; 7.00 P.M. Meeting, Chipman Hall, Tremont Temple, Boston

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*DESIGNERS SECTION — WEDNESDAY, NOVEMBER 8*

6.00 P.M. Society Rooms, 715 Tremont Temple, Boston

"Some Observations on Current Development in Foundation Engineering, Soil Mechanics and Hydraulics in Europe," by Dr. Arthur Casagrande, Harvard University

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# IMPORTANT ANNOUNCEMENT

THE Engineering Societies of Boston, of which the Boston Society of Civil Engineers is a member organization, has arranged to publish a WEEKLY JOURNAL announcing the meetings of the thirteen affiliated groups and containing interesting items about the activities of the Engineering Societies of Boston and of the individual societies.

This new publication will serve a number of purposes: (1) it will keep the more than 4,000 members of the Engineering Societies of Boston posted as to the meetings of *all* the individual societies; (2) it will provide the members of each society with *up-to-the-minute* information as to the meetings and other activities of that group; and (3) it will result in a saving of expense to the member societies as it will eliminate the necessity of sending out separate notices by each society.

The Boston Society of Civil Engineers wishes to co-operate to the fullest extent with the Engineering Societies of Boston in carrying out this new policy. Our members are urged to watch this JOURNAL for notices of meetings of the Society and of the Sections. Attention is called to the announcement of *"Coming Meetings,"* which will appear regularly in the B. S. C. E. JOURNAL. This advance announcement, together with the WEEKLY JOURNAL, will undoubtedly provide our members with an improved service.

ARTHUR W. DEAN, *President.*

JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume XX. OCTOBER, 1933 Number 8

# FOUNDATIONS FOR GEORGE EASTMAN LABORATORY OF PHYSICS AND CHEMISTRY, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

By Harry E. Sawtell, Member, Boston Society of Civil Engineers *

The foundation problem of the new George Eastman Laboratory of Physics and Chemistry, Massachusetts Institute of Technology, was directly concerned with the soil problem and tied back to the work on foundations for the buildings of the original group built in 1915.

## General Soil Problem

The varied character of the soil at the site of the Massachusetts Institute of Technology was described in considerable detail in the Journal of the Boston Society of Civil Engineers, Vol. V, No. 1, January, 1918.

Briefly stated, — bed rock is 120 to 135 feet below the general surface. Above the slate bed rock is found, an average of 15 feet of boulder clay, 60 to 80 feet of clay, and above most portions of the clay is a glacial deposit of sand and gravel varying in thickness from 35 feet near Massachusetts Avenue to nothing in other parts of the site. Above the glacial deposit or clay is a thick deposit of silt, muck and filling, found to vary from a few feet in thickness to 35 feet.

The thick clay bed for the most part is made up of what is called "medium" clay in accordance with ordinary classifications. In a few places "soft" clay was found, notably under the central or Administra-

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* Member of the firm of Chas. T. Main, Inc., Consulting Engineers, 201 Devonshire Street, Boston, Mass.

tion building. The "glacial gravel" consists of all grades of sand and gravel. Peat was also found in some parts of the site.

When the foundations of these buildings were designed in 1914–15, one of the fixed objects in the design was to support the piles in the glacial sand or gravel if it was thick and compact enough to take the loading. The report describing the design, published in the JOURNAL of the Boston Society of Civil Engineers, Vol. V, No. 1, pages 7 and 8, contains the following: "It has been assumed, however, that if settlement occurs it will be in the clay, and in order to reduce this settlement to a minimum, it was decided to spread the building loads over the glacial gravel as much as practicable, which in turn would still further distribute the loads over the clay bed." One of the rules governing the pile driving for the original buildings was: "Do not drive piles through the 'drift' into the clay, as the bearing power must be obtained in the stratum of glacial drift if possible." Where this glacial deposit was found to be too thin or disappeared altogether, it became necessary to depend directly on the clay for support.

In some cases the sand and gravel were found under a part of a building only, requiring the foundations to be designed to suit both kinds of soils, having different values and reactions under applied loads.

The permanent water level is very near the surface and the unstable fill is very thick. The surfaces of the glacial gravel or clay were at great and varying depths below both the top surface and the water level. The economical solution of the foundation problem was, therefore, the use of many piles having small loads.

Small pile loads were used throughout, and those piles which were driven into the clay for their support were given even smaller loads than those which could be stopped in the sand or gravel. This was done to equalize to some extent the resulting pressures on the subsoil clay, the gravel spreading the load on the clay when it sustained the loads of the piles.

In the original design of 1915 a definite effort was made to increase the load on the subsoil clay as little as possible by spreading the loads so as to make them as uniform as practicable. It was recognized at that time that any marked settlements occurring in these buildings would be due to the compression of the deep clay bed. That settlements have not been more uniform is due to the difference in character and depth of the clay and not to the use of piles driven into the clay.

Under the deep gravel beds near Massachusetts Avenue the clay is stiffer and of less depth; therefore settlements are, and will be, less. Under the central or Administration building the glacial gravel was

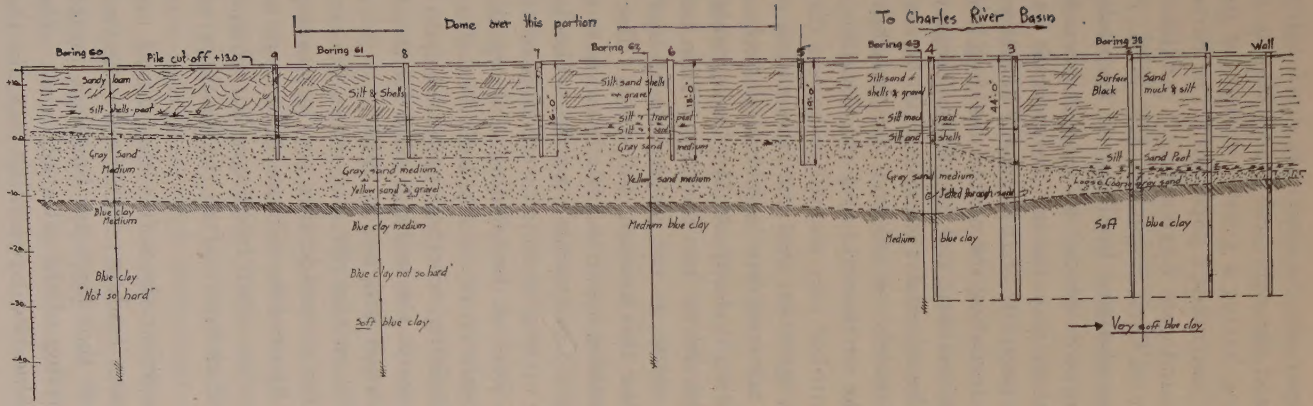

FIG. 1. — SECTION THROUGH BORINGS 60–61–62–63–38

Showing soil and lengths of piles

thinner, the clay deeper and its character much softer. In fact, under this building was found the only bed of soft clay in the site, which accounts for the building having a greater settlement than others.

In Mr. Arthur Casagrande's paper on "The Structure of Clay and Its Importance in Foundation Engineering," published in the JOURNAL of the Boston Society of Civil Engineers, Vol. XIX, No. 4, April, 1932, a profile of the Massachusetts Institute of Technology buildings was shown as "corroborative evidence" to support his contention that piles driven into undisturbed clay caused an increase in the settlements. This profile (Fig. 23, page 197 of that paper) is taken on the courtyard face of the buildings, including the central building and others. For the clay under the central building and for the buildings to the right of the center of the figure, the statement is made that the clay is "fairly homogeneous and approximately of uniform compressibility." In general, he concludes that the settlements and the location of the long piles driven into the undisturbed clay seemed to have a direct relation.

The writer finds, however, that the information on which this conclusion was based is not complete in several respects, as follows:

The central building is shown on Mr. Casagrande's profile as being supported by long piles driven into the clay. This, however, is not true for the building *as a whole*, as the accompanying profile (Fig. 1 of this paper) of the soil under this building will show. The profile submitted is taken on an axis running north and south and is at right angles to the view in Fig. 23 of Mr. Casagrande's article. The largest part of the building is supported upon piles which were stopped in the thick glacial sand bed back of the front of the building, and not as shown in Fig. 23.

There is no evidence of any marked difference in settlement of the various parts of the building, whether supported upon sand or clay, and it is a fact that the attempt was made to allow for the difference in action of the two materials when designed in 1914.

Fig. 23 does not show that the clay under the central building was soft or "very soft" for a considerable part of its depth, which is the case, however. The figure does indicate that there was a greater load per square foot than for other buildings, but this extra loading was largely added by the architect after the footings were designed and placed.

In the writer's opinion the softer clay found under this building together with the greater loading per pile were the causes of the greater settlements found over that for other buildings in the group, and was not caused by the driving of the piles into clay. The driving of piles into typical Boston blue clay appears to remould the clay very little, and causes little, if any, added settlements over those which may be expected from directly applied loads on such material if proper care is taken when applying them.

## EASTMAN LABORATORY SITE

The settlement record of the three buildings surrounding the new Physics and Chemistry building, named the George Eastman Laboratory, is interesting in that it tends to substantiate the statement made just above. The site of the Eastman Laboratory is surrounded on three sides by existing buildings, two of which are a part of the original group built in 1915 and the third (Mining) was built somewhat later.

Continuous records of settlements have been kept of the two buildings in the original group, but no record was made of the settlement of the Mining building because it was not located on the main court.

Briefly, the foundations of the three buildings are as follows:

The building to the south of the new Eastman Laboratory (Building No. 2) was supported on long piles driven into the medium clay for support as the glacial sand was too thin to be relied upon. The building to the west (Building No. 4, about 83 feet away) was supported on piles which were driven into the clay for the southern half, and supported in the sand for the northern half. The Mining building, to the north, was supported on piles all of which were stopped in the glacial sand. There is no building to the east of the new laboratory.

The three buildings all have different methods of pile support, and the settlement records will therefore be interesting. Fig. 2 shows the settlement records of the two buildings in the original group, and from the curves it will be found that Building No. 2 seems to have come to rest at about a 6-inch settlement, while Building No. 4 seems to have come to rest at a settlement of about 5¾ inches. The Mining building, on the north, when surveyed was found to have settled an equal amount, so that we had the very satisfactory condition of having all floors with which we had to connect on practically the same level.

This fact tends to prove that the varying condition of soil and their differing actions were reasonably well understood and provided for in the original design. It also seems to confirm the writer's opinion that when care is taken to give the clay practically the same loading it makes little difference whether the loads are applied at the surface (by means of the sand) or below the surface (by means of well-embedded piles), disregarding some minor contingencies. In other words, the driving of piles into this clay did not remould it to any extent nor cause excessive settlement for that reason.

Experienced foundation engineers knew at the time these foundations were designed that clay could be changed in characteristics by puddling, and that soft to medium clay would slowly subside under the loading ordinarily allowed and used. It has been known for many years that clay would consolidate to unknown amounts under the application of loads, and come to rest after various periods of time.

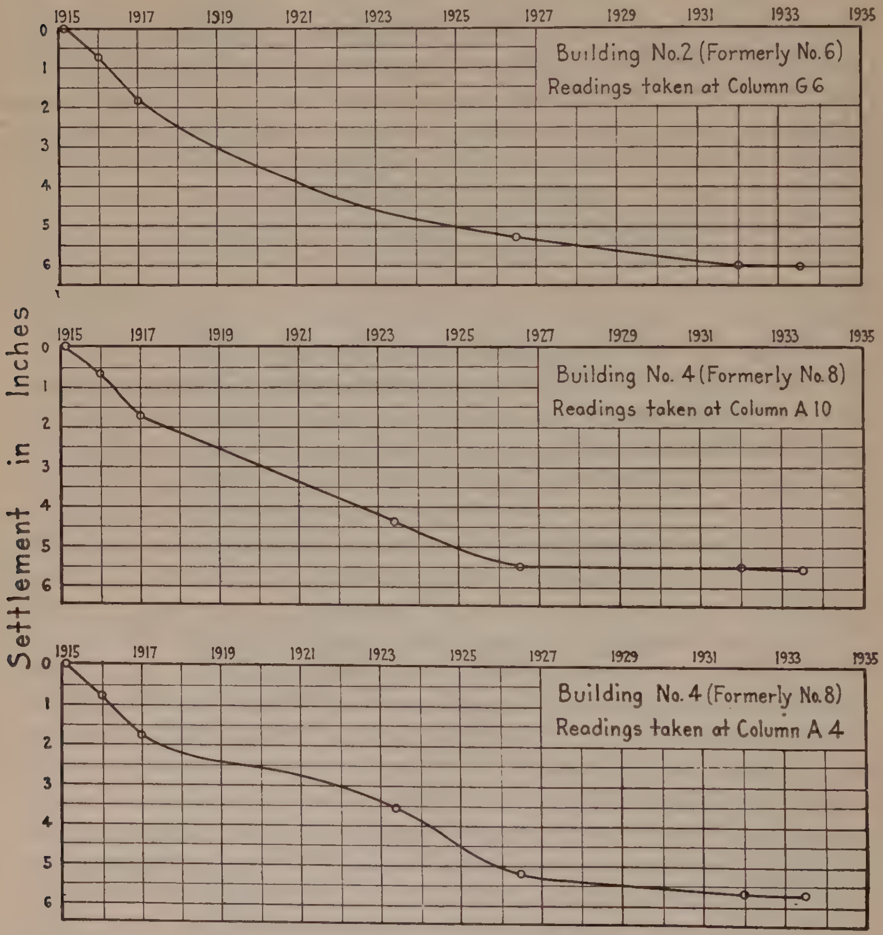

FIG. 2. — SETTLEMENT RECORDS

It is hoped that the new soil compression machines together with correct soil analyses will furnish data from which a better understanding may be obtained of the value and limitations of clay for supporting loads.

## EASTMAN LABORATORY FOUNDATIONS

Fig. 3 is a layout plan which shows the location of the new Eastman Laboratory (Building No. 6) with relation to the adjoining and near-by buildings (Nos. 2, 4 and 8). Fig. 4 shows three typical sections through piles and underlying soil. The piles were driven substantially as shown on this sketch. Fig. 5 shows a cross section at the southerly part of the building where the sand could not support the piles and long piles were driven into the clay.

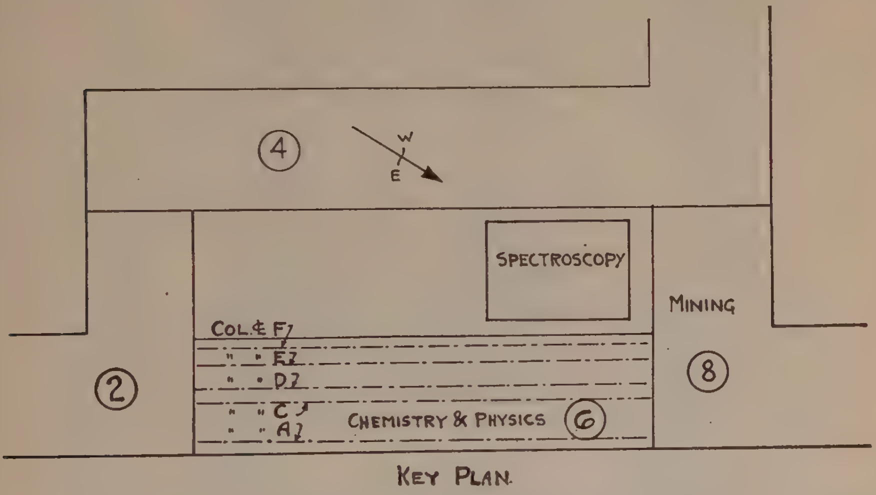

FIG. 3. — LAYOUT PLAN — NEW EASTMAN LABORATORY

It should be noted that the depth of silt and fill amounts to about 32 feet, and that the water level is well above the tops of the piles and approximately 22½ feet above the sand. The distance from the water surface to the tips of the piles averages about 48½ feet, constituting a hydraulic head. The thickness of the clay below the tips of piles to the boulder clay is about 47 feet of wholly undisturbed clay in which the consolidation will take place.

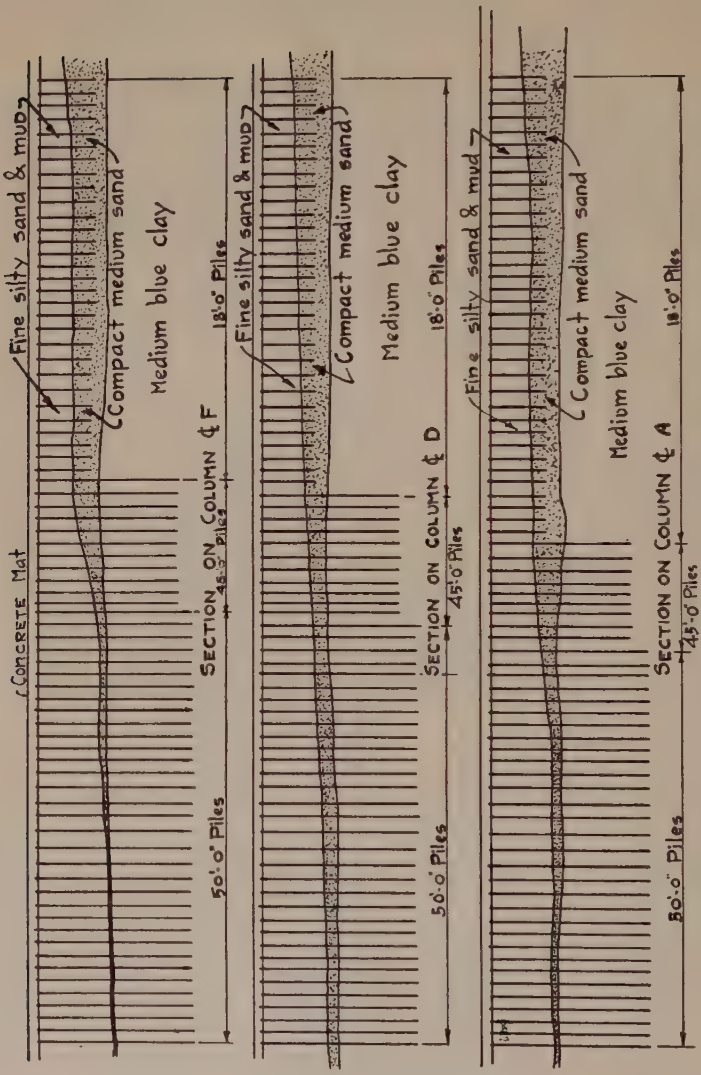

FIG. 4. — TYPICAL SECTIONS THROUGH PILES AND UNDERLYING SOIL

### ADDITIONAL LOAD ON CLAY

The average additional pressure applied to the clay at the tips of the piles is 1,500 pounds per square foot if full floor loads are used, but considering the fact that the loads were purposely assumed high, and that the floors will never be fully used at the same time, it is fair to state that the average additional load per square foot on the clay will be nearer 1,000 pounds.

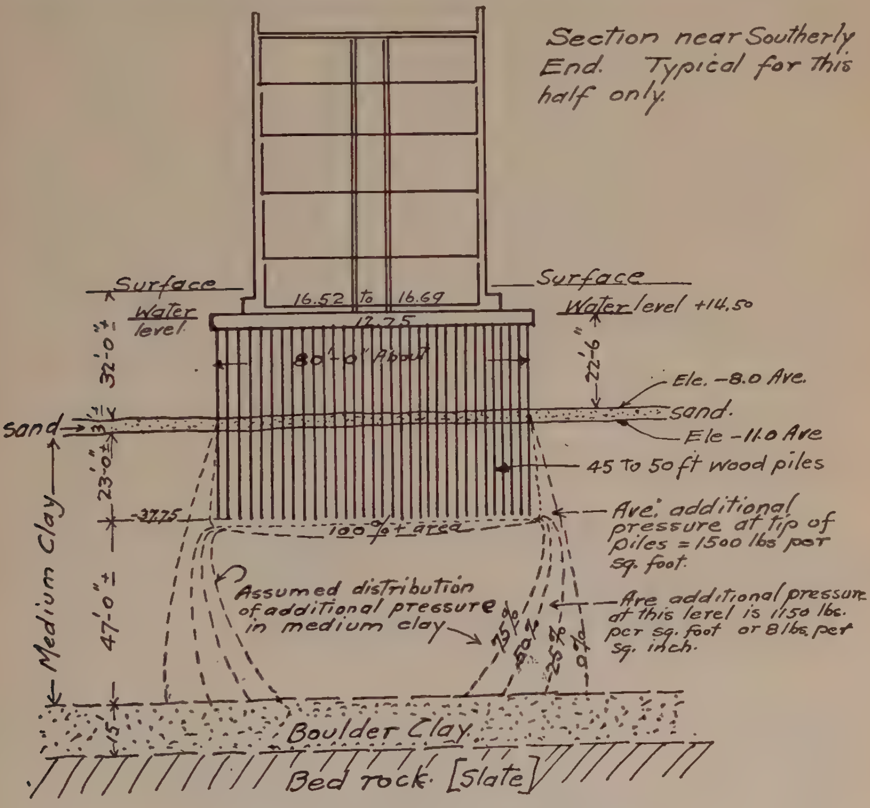
Section near Southerly End. Typical for this half only.
Surface Water level
Surface Water level +14.50
16.52 to 16.69
12.75
80'-0" About
32'-0" ±
22'-6"
Ele. -8.0 Ave.
sand
sand.
3'
Ele -11.0 Ave
45 to 50 ft wood piles
23'-0" ±
-37.75
100% ± area
Ave. additional pressure at tip of piles = 1500 lbs per sq. foot.
Medium Clay
47'-0" ±
Assumed distribution of additional pressure in medium clay
75%
50%
25%
0%
Ave additional pressure at this level is 1150 lbs. per sq. foot or 8 lbs. per sq. inch.
15'
Boulder Clay.
Bed rock. [Slate]

FIG. 5. — SECTION NEAR SOUTHERLY END

This pressure will not be increased directly under the piles mass, when considering the "pressure bulb," as much as would be the case if the material were sand, because the clay being much more plastic will not bunch up under the center as would sand grains. Clay and water will flow laterally, thus lowering the pressure at the center and permitting it to be more uniform than it could possibly become in sand. This relief or lowering of the concentrated pressure at the center causes the greatest settlements to take place in the early years of the structure.

The pressure bulb under the pile mass is of less width in clay than would be the case in sand, but there undoubtedly is a bulb formed and it shows that the additional building pressure becomes less as depth is attained. The result is that we have a constantly decreasing additional building pressure on the clay as we approach the boulder clay and bed rock below.

This reduced building loading on the clay must be taken into account when using the results of compression tests to determine the probable settlements of a superstructure placed on the clay. To take the maximum settlement or compression obtained from a sample of clay and multiply it by the depth of clay will give an answer far in excess of the actual settlement. On the other hand, if an average pressure (taken from the pressure bulb) can be determined and the amount of settlement for that additional load found, and allowance made for the actual resistances as noted in the second paragraph below, the result will be nearer to the actual settlement of the structure.

Buildings adjacent to the new laboratory, constructed along these same lines, have shown that they have practically come to rest at about a total settlement of 6 inches.

Referring to Fig. 5 it can be seen that the de-watering of the clay immediately under the piles (where the pressure is greatest) must take place largely by lateral flow, and considering (*a*) the width of this pile mass, (*b*) the depth of the tips below the surface of the clay, (*c*) the pressure of sand and fill above the clay, (*d*) the hydraulic head of 48.5 feet and (*e*) the light additional loading, it may be assumed that de-watering will not take place to any great extent, and that the structure of this medium clay will easily support the loads without serious settlements. For most of the southern half of the building the piles are spaced closely and are given a loading of about 7 tons each.

An economical *design* is one where there are just enough piles to distribute the load uniformly to the highest stratum capable of carrying the total load. Each pile corresponds to a small isolated footing or unit, as shown in Fig. 6. The most economical *spacing* of piles is one in which the units (pressure bulbs) are just sufficient to produce bulbs which at intersections are practically tangent to each other, thus using up the entire area at this depth. (See Fig. 6.) If piles are spaced too far apart the pressure will vary in the clay at the tips, being practically nothing in the space between the piles, thus affording no added resistance to an upward movement of the clay.

The best *loading* for this kind of clay is the one where the piles are reasonably close together and carrying a small load, to be transmitted to the clay, thus producing a low pressure practically uniform in its intensity at the tips of the piles. Large loads on piles or caissons more widely spaced (in clay) produce concentrated loads whose maximum pressures are too great for best results. Piling was thus used in this foundation in such a way as to cause as little general settlement or mass deformation as possible. Deeply embedded piles carry the loads to levels well under the surface of the clay, and this reduces the rate of de-watering in the sub-clay. The surface friction on the piles in the clay is kept small (200 to 300 pounds per square foot), which prevents the possibility of shearing action under working loads.

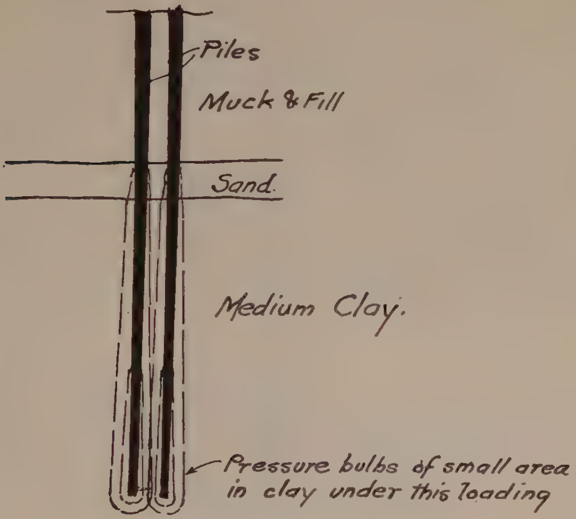

FIG. 6

The driving of the piles by steam hammers into this medium clay apparently results in a pushing action, and the rapid action of the steam hammer seems to have kept the film of watery clay, which accumulates at the surface of the pile, in a continually watery state while driving, allowing some water to pass up around the pile to the surface of the clay, thus relieving some of the water pressure below. This action aided in preventing very much remoulding of the clay around the pile.

Driving the new piles adjacent to the original building (No. 2) developed an interesting phenomenon. The new piles are probably about 5 feet deeper in the clay than the older piles under Building No. 2, and after the driving of the new piles it was found that the wall and near-by corridors had been raised about $^{3}/_{16}$ of an inch. After the interval of about two years we find that Building No. 2 has returned to the original level and has developed no further settlement. A heaving of the ground surface also occurred around the new piles when driving, but not as much as was expected, and it did not equal the displacement of the piles. The action indicates that the clay around the new piles was not appreciably remoulded but was compressed to such an extent as to exert a positive pressure on the clay around the old piles, and that the old clay was of sufficient stability and density to transmit the upward component of the force to the old piles and to raise the building slightly. Such favorable action would not have occurred if the piles had not been deeply embedded and properly spaced.

The deformation of the clay around and between the piles was such that the elasticity of the clay was not destroyed, as witnessed by the quick gripping of the piles when driving was stopped and as shown by the redriving tests on piles well embedded in the clay. No action comparable to remoulding the clay took place. (See Table I.)

TABLE I. — REDRIVEN WOODEN PILES

| PILE (No.) | Muck (Feet) | Sand (Feet) | Clay (Feet) | First Driving Value (Tons) | Rest (Hours) | Redriving Value (Tons) | Gain (Per Cent) |
|---|---|---|---|---|---|---|---|
| 22F3 . . | 13 | 6 | 22 | 10.4 | 0.5 | 16.6 | 60 |
| 8G8 . . | 19 | 3 | 27 | 8.3 | 0.5 | 25.8 | 201 |
| 16Q7 . . | 12 | 6 | 26 | 9.4 | 0.75 | 25.0 | 166 |
| 10M11 . . | 15 | 6 | 25 | 6.0 | 0.75 | 20.7 | 245 |
| 16F7 . . | 13 | 6 | 23 | 7.2 | 15.5 | 22.4 | 210 |
| 16J18 . . | 13 | 6 | 23 | 8.3 | 15.5 | 24.0 | 190 |
| 22F11 . . | 13 | 6 | 23 | 10.4 | 15.5 | 30.0 | 187 |
| 257 . . . | 19 | 2 | 29 | 10.2 | 15.5 | 23.3 | 127 |
| 10D3 . . | 17 | 5 | 25 | 7.1 | 15.5 | 39.3 | 450 |
| 16B7 . . | 13 | 6 | 23 | 8.3 | 32.0 | 33.3 | 300 |
| 25B2 . . | – | 5 | – | 23.5 | 0.5 | 30.2 | 29 |
| 25B1 . . | – | 6 | – | 30.0 | 0.5 | 31.1 | 4 |

## Redriving Tests

Piles allowed to rest one-half hour gained 60 per cent or more in formula value. Those having a three-quarters hour rest gained 166 per cent in value, while those having fifteen and one-half hours' rest gained about 200 per cent in value, on the average. Driven into "soft" clay the gain is generally in greater percentages.

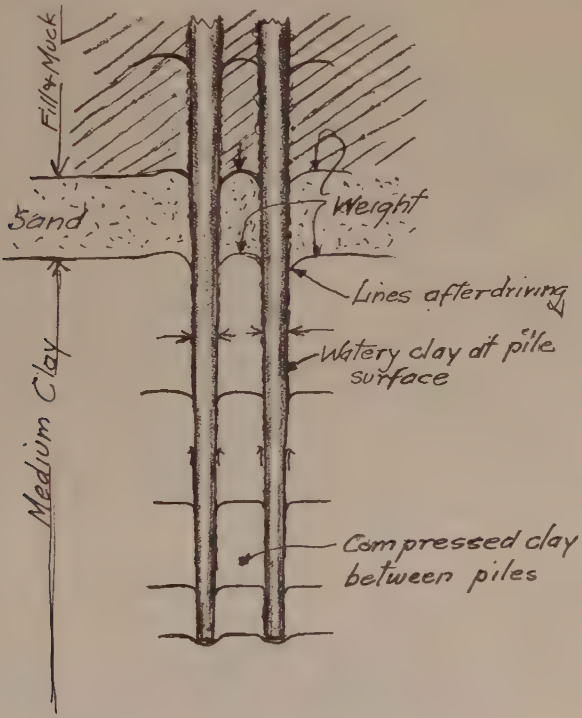

FIG. 7

Fig. 7 shows in a general way the action of the soils at driving. The clay (after driving stopped) immediately tended to straighten out from pressure above and from its compressed and deformed state, thus forcing good original clay back to the surface of the pile. The pickup in strength cannot be wholly accounted for in any other way. Remoulded clay does not, or will not, have the same gripping strength as exhibited here. The small amount of remoulding which does take place at the surface of the piles must also act in a manner like the process of coagulation which, combined with the action described above, gives results shown in Table I.

When driving piles, the laminated structure of the clay is bent down near the surface of the pile so that a redriving test gives a better idea of the gain in value of skin friction than a pulling test, because in a pulling test a heavy suction resistance will be obtained for the whole of a tapered pile or under the point of a straight pile. The redriving tests indicate the gain in skin friction and resistance to sudden application of load. The gain in skin friction may not tell the story of frictional resistance of the pile under long applications of load, nor the story of the deformation of the surrounding clay under the load applied for years, but the answer to this may be found in the curve of settlements of adjacent buildings, where it was also necessary to support them on piles deeply embedded in medium clay. This long-time action of piles in clay was known and the reduction of settlements for these buildings was attempted by the use of long piles, small loads and as wide a distribution of loading as could be obtained.

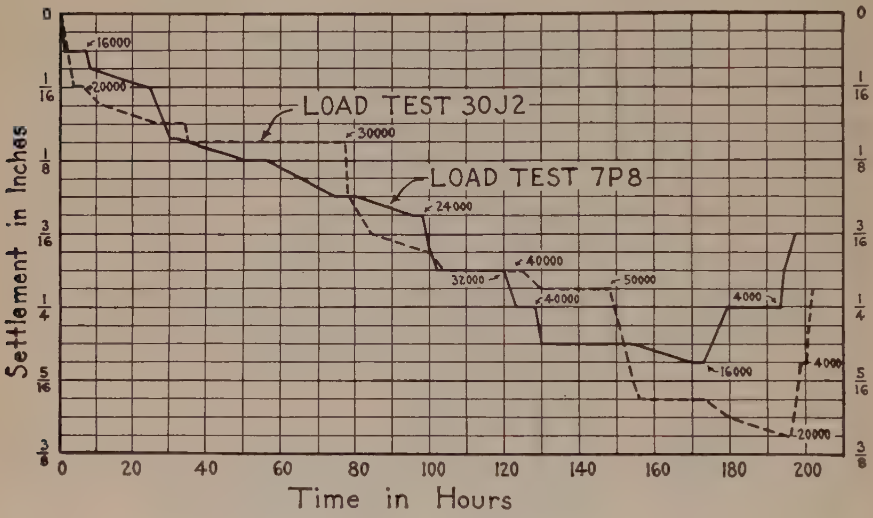

FIG. 8. — RESULT OF LOAD TESTS

## LOAD TESTS ON PILES

Fig. 8 shows the result of load tests on Pile 30J2 which was driven into and stopped in sand; and on Pile 7P8 which was driven through the stratum of sand not capable of supporting it into the clay 22.7 feet. The latter pile was somewhat disturbed by the driving of other piles during the test, but showed good characteristics. The recovery during the unloading of this pile should be noted. The load tests on these piles were made to determine the frictional resistance at the pile surface under fairly long-time application of load.

To know the frictional resistance or grip of a soil on a pile, even if it is a part of a group, is to gain knowledge of the ability to transfer the load applied to it out into the clay around it. These piles are to act in groups and not as single piles, therefore skin friction is of prime importance. This action is distinct from that of the action of the group as a whole. The loading selected for a pile driven into clay is actually arrived at by other considerations, such as the least possible settlements in the clay as a whole, character of clay, etc., as previously stated.

Building laws frequently allow four tons of load to be applied to stiff clay, but the additional loading given this medium clay at the Massachusetts Institute of Technology site averages less than ¾ ton per square foot, or 14 pounds per square inch. The medium clay at this site will undoubtedly carry this load with a very moderate settlement.

## SUMMARY

The results of tests, including driving, redriving, loading and unloading, all confirm the statement that soils are weak solids having elastic properties which must be considered in foundation design. The loading selected for this foundation appears to be well within the elastic limit of this clay. Piles driven into this clay seem to remould it but little, and not enough to cause additional settlements.

The Engineering News formula was used during the driving of these piles, but was used as "gauge" or "yard stick" only. The weaknesses and limits of this formula are quite well known, which is a large advantage in its use. Other formulæ will have their limits, as no formula can be made which will apply to all of the soil combinations and driving differences with perfect results.

Piles driven through sand and deeply bedded in this clay were better supporting piles than when placed in clay alone. The writer sees no objections to driving piles through sand into clay provided conditions are all met and provided for, one of which is proper embedment into the clay.

The weight of the sand on the clay resists the excessive vertical deformation of the clay and assists in the quick gripping of the pile when driving is stopped. The sand also steadies the pile during driving to some extent, and thus reduces the remoulding tendency in the clay near the pile due to vibration during driving.

The clay surrounding the piles, in this job, is undoubtedly in better condition after driving than previous to driving, as it probably has less water content, is somewhat compressed, practically undisturbed and deformed but slightly, especially at the tips of the piles.

The additional building load has been transferred to a practically undisturbed bed of clay at a good distance below the surface of the clay, and the bed of clay below the tips wherein settlement will take place has been very much reduced in depth, as indicated in Fig. 5.

# PENETRATION MACADAM AT LOW COST

By R. W. Coburn, Member, Boston Society of Civil Engineers *

(Paper presented before the Association of Highway Officials of the North Atlantic States at Atlantic City, N. J., February 16, 1933)

I am glad to have an opportunity to discuss this subject, for I have a great deal of confidence in a properly designed and carefully built penetration macadam road. This type of construction, more commonly called bituminous macadam, has always been very popular in Massachusetts on account of its moderate construction cost and its low maintenance cost. At the present time more than 60 per cent of the mileage of the high type modern roads in the State, both on State highways and in cities and towns, are of this construction.

Some of the members present may not agree with me when I classify bituminous macadam as a high type modern pavement, but if that is so, I believe it is because they have in their mind an entirely different pavement than the one I am discussing. The phrase "Bituminous Macadam Road" has undoubtedly been very much abused, for many people use it or think of it when they refer to a water-bound macadam road or any road that has been surface treated with any bituminous material. My idea of a bituminous macadam road is a modern high type pavement, and it is my belief that it has a wide field for use wherever trap rock or other suitable stone is available. Furthermore, I believe that in many places it can be built and maintained more economically than any other type of pavement.

## History of Bituminous Macadam Roads

Before going into the details of construction, it will be well to define what is meant by a "Bituminous Macadam Road," and to consider briefly its history. The definition adopted by the Association for Standardizing Paving Specifications is as follows: "Bituminous macadam pavements are those consisting of broken stone and bituminous materials incorporated together by penetration methods."

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* Construction Engineer, Massachusetts Department of Public Works, Boston, Mass.

The modern bituminous macadam road is comparatively new, having come into popular use during the last twenty years. Its popularity was due to the fact that the automobile was fast destroying the old water-bound macadam roads, and a binder other than dust and water was necessary. To Massachusetts, perhaps more than to any other State, belongs the credit for pioneer work in this type, and it may be considered pioneer work, for this type was still in its experimental stages up to 1912, at which time the Massachusetts standard type of penetration was developed. It can be safely said that up to this time heavy asphalts had not been sprayed by distributors under pressure. Lighter materials had been used and the work was done by hand or other crude methods.

The experimental or pioneer work mentioned above was carried on from 1907 to 1911, inclusive, each year showing some new development or improvement. At first the bituminous materials were used to preserve the old water-bound macadam roads and to lay the dust, but it was soon found that better results could be obtained if the bituminous binder was incorporated into the road itself.

One of the earlier types of bituminous macadam developed in Massachusetts was the bituminous macadam hot oil type. This is not considered a modern pavement today, and is no longer being built, but there are still a few miles of it under maintenance in the State which were built previous to 1912. It was constructed by spraying 90 per cent asphaltic oil on top of a water-bound macadam base, and then spreading about 2 inches of 1½-inch stone on top of the oil blanket and rolling the stone into it. A second application of the same oil was then sprayed into this 1½-inch stone, covered with pea stone or sand and thoroughly rolled. Sometimes, if necessary, a light seal coat was added.

## Construction of Standard Type

The bituminous macadam that I am going to discuss in this paper is a type that was developed in Massachusetts in 1912. It consists of a 2 to 3 inches top or wearing surface of 2½-inch stone penetrated with heavy asphalt or tar. This top surface is usually laid on a prepared foundation consisting of bank gravel and a base course of broken stone, but in many places, especially in resurfacing work or on a secondary road, it is possible to omit the macadam base course and lay the 2½-inch bituminous macadam surface directly on top of an old macadam road or gravel foundation.

In discussing this subject I shall treat it briefly under the following headings:

(1) Materials used in construction.
(2) Machinery and equipment necessary.
(3) Inspection of the work.
(4) Design and method of construction.

## *Materials*

The materials necessary for the construction of a good bituminous macadam road are, first, a good hard and tough stone, and second, a good grade of bituminous material, either asphalt or tar of the proper consistency.

The stone for the penetrated top should preferably be a good commercial trap rock or a local stone of its equivalent, having a French coefficient of wear of not less than 14 and a toughness of not less than 12. However, where the work is to be done on secondary roads, and a local stone of 12 and 10 is available at a lower cost, it can be safely used with satisfactory results. The base course stone can be of an inferior quality, 10 and 8 being the usual specification, but 8 and 8 would be satisfactory on secondary roads.

The bituminous material may be any one of three kinds which shall have certain prescribed properties, and, among other requirements, must meet the following:

(A) Fluxed native lake asphalts shall have a penetration of 120 to 150.

(B) Oil asphalts shall have a penetration of 85 to 120.

(C) Tars shall have a consistency of 120 to 180 by the Float Test.

## *Machinery and Equipment*

The amount of machinery and equipment necessary for this work is probably smaller than for any other type of modern pavement. For that reason it is popular in cities and towns where they want to do the work themselves and where they do not own expensive machinery. Provided the stone is trucked to the job and the bituminous material sent out from a heating plant in a truck sprayer, about the only machinery needed on the job is the steam roller.

### *Inspection*

The inspection in the field is a most important phase of this work, and it is my opinion that more penetration roads fail because of poor workmanship and poor inspection than from faulty materials. A bituminous macadam road may fail from poor design, because foundations and drainage were omitted where they were needed; or it may fail because of poor quality of bitumen, or because work was done when weather conditions were unsatisfactory; or it may be caused by too little or too much bitumen; but I believe that the majority of the failures could be prevented if the inspectors were properly trained in the details of the work. Any man who wants to be successful on this kind of work should leave off his white collar and good clothes. The work is dirty, and no man can see what is going on unless he is willing to dress so that he can get into the midst of it.

### *Method of Construction*

The first step in the construction of the penetration road is the provision for adequate drainage. In loamy or clayey soils, a subgrade of granular material (bank-run gravel) should be laid, or a stone base on top of about 4 inches of sandy gravel. In wet and springy ground, side drains should be built or ditches should be provided along the sides of the road in order to keep the ground water out of the subgrade. The elimination of water in the subgrade is very important, and unless bleeders of porous material are built at regular intervals to carry the water out from under the pavement, the subgrade may become saturated and lose its normal bearing power, and in cold weather cause heaving or frost action. These bleeders are especially important at low points in the profile, and here they should be built on both sides of the road to make certain that there is no ponding of water under the pavement. Having drained the road and provided suitable subgrade and shoulders, the next step is to roll the subgrade thoroughly with a heavy roller until no settlement can be discerned.

*Base Course.* — Upon this compact subgrade there is then spread a layer of broken stone, preferably No. 1 stone, but if it is necessary to use the output of a crusher, then the No. 1 and No. 2 stone may be mixed, being careful that they are mixed uniformly and that there is not an excess of the No. 2 stone. This layer should be carefully spread either from self-spreading trucks or with stone spreaders, and after rolling should have a perfect cross section 2 to 2½ inches below the desired finished grade. Up to this point no dust or sand binder should have been added, for it is very important to get the maximum amount of mechanical bond before adding the binder or filler, and it is only when the roller will no longer leave its marks on the surface of the broken stone that the stone can be considered thoroughly rolled. In spreading the binder, it should be applied in two or three applications, while the roller is working, and in no case should it be dumped on the surface of the broken stone. During the rolling of the base course it is quite important to roll out on the shoulders for 12 to 18 inches in order to insure a strong compact shoulder that will not weaken or settle when the final rolling is being done on the finished surface. After the base course has been bound up with the binder and rolled until it is thoroughly compacted, it should resemble somewhat a water-bound macadam surface, except that it is not bound up so tightly. The surplus binder is then broomed from the surface and the top or wearing course can be spread.

*Top Course.* — The best results will be obtained in the top course if No. 1 trap rock is used, being careful that there is no No. 2 or finer stone mixed with it. This trap rock should be spread with a mechanical spreader or from dumping boards with forks, and the dust, dirt and small stone left on the boards thrown to one side and wasted. This is important, for with almost every truckload of broken stone there is a bushel or more of dirt and small stones, and if this is left in the road in one spot, as is likely, proper penetration will not be obtained when the bitumen is applied, as the voids will be partly filled with dirt. Furthermore, there would probably be a blot or "fat place" on the surface where the bitumen could not penetrate. Having spread the top stone carefully, and having done all the patching that can be seen at the time, the top surface should be lightly and carefully rolled, and the patching continued until the road surface looks absolutely perfect. The patching should be done with No. 1 stone only, and it is important to do it before the surface is rolled too much, as it is very hard to roll a patch into a well-rolled road. The top course is usually 2½ inches thick. When the stone is so rolled that the roller does not make a track on the stone, but before the stone begins to break up under the roller, the top course is ready for the bituminous material.

*Penetration.* — The engineer in charge of the bituminous work should now carefully inspect every square foot of the surface where he intends to apply the asphalt, to see that there are no depressions and that the stone is clean and of the proper depth. Special care should be taken to see that the edges of the road are straight and neat, and that dirt from the shoulders has not been allowed to enter the top stone, for if the edges are neglected, they will be the first to ravel. He should then see that the bituminous material is of the proper temperature and that all the nozzles of the sprayer are absolutely clean. Having looked after the details, word may be given to go ahead and apply the bitumen. If asphalt is used, it should be heated to a temperature of about 350 degrees and applied under a pressure of 60 pounds per square inch, at a rate of two gallons per square yard, either with a motor-distributor or with a wagon sprayer, but not with a single nozzle. If the distributor streaks and does not apply the asphalt uniformly, it should be stopped at once, and either removed from the top stone while the nozzles are being cleaned, or a trough should be put under the nozzles while they are dripping and being cleaned.

After the load has been distributed on the road, and before the asphalt is covered with pea stone, the engineer should carefully examine the surface to make sure that there are no white streaks or spots that have been missed. If any are found, they should be carefully patched with a pouring can. Having examined the surface and found it O. K., ¾-inch pea stone should at once be uniformly and lightly thrown upon the asphalt and the surface should be rolled while warm. The application of pea stone should be so light that the roller at all times will be bearing on the No. 1 stone and not on a cushion of pea stone. If the wheels of the roller stick to the asphalt, as they probably will in warm weather, they should be oiled. More pea stone should be added as the rolling takes place, until the surface has an even texture, with the voids mostly filled, but with no loose or surplus pea stone on the surface. Having covered the first application of asphalt with the proper amount of pea stone, and having thoroughly rolled the surface, then the surface should be swept clean of all surplus pea stone and the seal coat applied.

*Seal Coat.* — The seal coat should be applied at a rate of not over one-half gallon per square yard. It is even more important than with the penetration work that the asphalt be hot and the nozzles clean, for the distributor is moving about four times as fast while applying the seal coat. Tar paper or building paper should be spread down on the road and covered with sand, at the point where the distributor is to be opened up, and the distributor should start far enough back from the point where it is proposed to begin spraying so that it will have a flying start and be going at the proper speed when the nozzles are opened.

After the load has been applied, the asphalt which landed on the paper is wrapped up and thrown to one side, and if the work has been properly executed, there will be a straight line joint where the seal coat started, without a surplus of asphalt on the portion already sealed.

If the work has been properly done, the surface will not have a blot or fat spot on it, and I consider a fat spot a defect in the road. These fat spots may be caused by dirty stone, small stone, or stone spread too thin, or overlapping either at the center joint or at the beginning or end of a load, all of which can and should be avoided in order to get perfect results.

The seal coat is now covered with ½-inch pea stone, using about the same method as outlined above for the penetration course, that is, being careful not to put too much on at any one time, keeping it free from bunches, and keeping the roller going while the seal coat is still warm. This final covering of pea stone should consist of just a little more than will stick to the road; it should be broomed about uniformly, and thoroughly and carefully rolled, this final rolling being very important, for there is then no danger of breaking the No. 1 stone. A 12 to 15 ton roller is better than the 10 ton roller for this final work. After applying both the penetration work and the seal coat, the bituminous material that overlaps on the shoulders should be trimmed off to the line of the edges of the broken stone, in order to get a good, clean-looking line along the edges of the road.

I have been speaking, up to the present, of the asphalt penetration, for I believe that the asphalt penetration is, in most places, superior to the tar. However, they are both used in bituminous macadam roads, and the same methods for carrying on the work should be used with either material. Asphalt probably has a longer life than tar, and if properly used will require less maintenance. Tars, however, may be used with certain local stones where the travel is not too heavy, or in cold weather, with more success than asphalt, for tar will work its way into smaller voids than asphalt, and works down into the road with travel and hot weather, while asphalt works both upward and downward.

## Proper Season for Good Penetration

In the northern States the season of the year in which the work is done is very important. I do not believe that perfect results can be obtained on work done after the first of October. Work finished previous to August 1 or September 1 is usually much better than work done later in the season. The reason for this is that work done in the early summer gets the benefit of being rolled during the hot weather, and the travel and sun help to bond the stone and the asphalt.

### Bituminous Macadam for Resurfacing

Bituminous macadam has been used extensively and successfully to resurface the old water-bound macadam roads, and in this field it can be most economically used, for in many places the old macadam surface will furnish the necessary base for a new 2-inch wearing surface. When built in this way, care should be taken to locate the spots in the old road that show any signs of weakness in the spring or break with the frost, and they should be strengthened either by adding base stone, or by subgrading.

### Mileage and Cost of Work

Having described in detail how the bituminous macadam road is built in Massachusetts, and when and where it can most successfully and economically be used, let me turn to the economic side and give you some figures showing its mileage, its cost and its service record; and finally, from the data presented, I shall try to prove to you that this type of road has great possibilities for low-cost construction.

There are on the state highway system of Massachusetts today 922 miles of penetration roads, most of which are asphalt macadam. This mileage totals more than all other types combined, and during the last few years more than half of all the work supervised by the department has been of this type.

This type has been used almost exclusively on the more important state-aid roads, as well as on the main state highway routes, where trucking is not of major importance. The construction cost for this type has varied all the way from $8,000 to $35,000 per mile, depending on whether the project was resurfacing or new heavy construction, and depending further on the amount of grading and drainage, the width and depth of the pavement, the location of the project, and the year in which it was built.

On account of these wide variations in costs and conditions, I am going to take three or four typical projects, built ten to twenty years ago, and give you a record of their service, and then figure their present-day cost of construction, using average 1932 contract prices and assuming uniform conditions.

## EXPERIMENTAL ROAD

First, let me refer to an experimental project of bituminous macadam, built in 1913 between the cities of Lowell and Lawrence. This road, 18 feet wide and about 3¼ miles in length, was subgraded with from 6 to 12 inches of gravel, and the wearing surface, 2 inches in thickness, was laid on a 4-inch macadam base and supported by 3-foot gravel shoulders 6 inches in depth. Six sections, approximately each one-half mile in length, were built of asphalt, using five different kinds of oil asphalt and one native asphalt, making a total of three miles of asphalt macadam. The remainder, about one-fourth of a mile, was penetrated and sealed with tar.

The actual cost of this road when built, including all grading and drainage, was about $25,000 per mile, but excluding the drainage and the excavation, other than that necessary for a 9-inch gravel subgrade, the cost would have been about $16,000 per mile. However, using 1932 average contract prices, this $16,000 per mile would be reduced to about $9,500 per mile.

Now, let us look at the service record of this road. It has been under maintenance nineteen years, and at the present time is carrying more than 7,000 vehicles per day, 10 per cent of which are trucks. At no time have any of the asphalt sections been given a surface treatment, and from the maintenance engineer I have obtained the information that only $160 per mile was spent on the surface of this road last year, and that the average cost per mile per year since the road was built was less than $64.

## SECOND TYPICAL PROJECT

Now let me turn to a second typical project. This is on Blue Hill Avenue in the town of Milton, just outside of the Boston city limits and on the main automobile and trucking route from Boston to Taunton, Fall River and New Bedford. Here in 1924, 3½ miles of 2½-inch asphalt macadam was built on top of a foundation consisting of 3½ inches of broken stone and 6 to 12 inches of bank gravel. The width of the surface was 24 feet but the outer 2 feet on each side had no gravel foundation, as they were added as an afterthought to provide hardened shoulders. I have not available here the actual cost of this construction, but assuming an average of 9 inches of excavation, backfilled with a 9-inch gravel foundation, this 2½-inch penetration on top of a 3½-inch macadam base, would today cost, for a 24-foot width, including foundation for full width, about $12,500 per mile, which would be equivalent to $9,500 per mile for an 18-foot width.

This road is now carrying about 12,000 vehicles per day, 10 per cent of them being trucks, and the cost of the surface maintenance, both for the year 1931 and for the average of its eight years of life, has been less than one cent per square yard, the actual figures being $170 per mile.

## Bituminous Macadam on Stone Fill

For a third typical project I have picked out a road in the central part of the State, on the old Boston Post Road, built with a stone fill foundation, in the town of Spencer. Here, in 1921, 3 miles of asphalt macadam were built, the design calling for a 2½-inch penetration on top of a base course of 2 inches of broken stone over a 10 inch stone fill and 4 inches of sandy gravel. Here again I have not got available the original cost of the project, but assuming 12 inches of excavation for a 20-foot width, backfilled with the 4-inch gravel subbase and the 10-inch stone fill, and surfaced with the two layers of broken stone, the top layer being penetrated with asphalt, and supported with gravel shoulders 3 feet wide and 6 inches deep, this road, at 1932 average contract prices, would cost about $13,000 per mile for 20-foot width, equivalent to $11,600 per mile for 18-foot width.

This section of road, now under maintenance eleven years, is at present carrying traffic of about 6,000 vehicles per day, with a surface maintenance cost for the year 1931 of one-half cent per square yard, and with an average cost per square yard per year of six-tenths of a cent.

## 2½-Inch Penetration on Gravel Base

For a fourth and last typical project I have picked out a section of road built in 1913, where a 2½-inch tar macadam top, 18 feet wide, was placed directly on top of an old gravel road. This project is in the town of Methuen and is adjacent to the experimental road previously described in Dracut. The section, originally built, was 1¼ miles in length, but a small portion has since been widened and topped with a second course of penetration. The cost of this construction in 1913 was about $8,000 per mile, more than half of the value representing the cost of the tar in the road. The cost of doing the same work today, using asphalt instead of tar, would be slightly less than $5,000 per mile.

Now let us look at the service record of this low-cost, tar-penetrated macadam road. It has been under maintenance nineteen years, and at the present time is carrying more than 7,000 vehicles per day. The maintenance figures show that over the entire nineteen years the average cost per mile per year has been $250, but for the last two years it has averaged only $50 per mile. The reason for this paradox is that this tar macadam has been given several surface treatments, the last of which was in 1928, and these have caused the average cost per mile since built to be high, but has caused the cost per mile for the year or two following the treatment to be low.

## CONCLUSION

In closing this paper I should like to draw a few conclusions from the data presented in the above projects and outline for you a design for building bituminous macadam at low cost. My conclusions are as follows:

(1) Bituminous asphalt macadam can be built as a high type pavement at moderate cost.

(2) With proper foundations, it will carry moderate to heavy travel with low maintenance cost.

(3) It will go fifteen to twenty years without a maintenance seal coat.

(4) It can be built at low cost to carry light to moderate travel.

My design for low-cost construction would be to build an 18-foot 2½-inch asphalt penetration surface, with 3-foot gravel shoulders, on top of a foundation of 10 inches of bank gravel, or a combination of 3 inches of bank gravel and 8 inches of stone fill. This design, on the basis of a profile grade 6 inches above the old road surface, and excluding all other grading and drainage, since such grading and drainage would be common to any type of surface, would cost today in Massachusetts, using 1932 average contract prices, from $6,000 to $8,600 per mile, depending on whether gravel or stone fill was used in the foundation.

This design, according to service records cited above, should carry from 1,000 to 5,000 vehicles per day, with an average annual surface maintenance cost over a period of fifteen years of not over $125 per mile. Furthermore, this type of construction could be topped at any time in the future with another 2-inch bituminous macadam surface without throwing away or discarding any part of the old road.

## Cost of Building Low Cost Road *

(Not including drainage — and assuming the new grade to be 6 inches above the old surface)

| | |
|---|---|
| Width of surface | 18 feet |
| Type of surface | 2½-inch macadam |
| Bituminous material | Asphalt, 2⅜ gals. per sq. yd. |
| Foundation | 10 inches of gravel |
| Shoulders | Gravel, 3 feet by 6 inches |

| | | | Per Mile |
|---|---|---|---|
| Excavation | 1,906 c.y. at | $0 20 = | $381 20 |
| Gravel borrow | 4,224 c.y. at | 28 = | 1,182 72 |
| Broken stone | 1,544 tons at | 1 66 = | 2,563 04 |
| Asphalt | 25,080 gals. at | 07 = | 1,755 60 |
| Fine grading | 14,080 s.y. at | 01 = | 140 80 |
| | | | $6,023 36 |

| | |
|---|---|
| Width of surface | 18 feet |
| Type of surface | 2½-inch macadam |
| Bituminous material | Asphalt, 2⅜ gals. per sq. yd. |
| Foundation | 8-inch stone fill over 3 inches of gravel |
| Shoulders | Gravel, 3 feet by 6 inches |

| | | | Per Mile |
|---|---|---|---|
| Excavation | 2,200 c.y. at | $0 20 = | $440 00 |
| Gravel borrow | 2,347 c.y. at | 28 = | 657 16 |
| Broken stone | 1,544 tons at | 1 66 = | 2,563 04 |
| Asphalt | 25,080 gals. at | 07 = | 1,755 60 |
| Fine grading | 14,080 s.y. at | 01 = | 140 80 |
| Stone fill | 2,347 c.y. at | 1 30 = | 3,051 10 |
| | | | $8,607 70 |

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* At 1932 average contract prices.

# OF GENERAL INTEREST

## PROCEEDINGS OF THE SOCIETY

## MINUTES OF MEETINGS

### Boston Society of Civil Engineers

SEPTEMBER 27, 1933. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club, and was called to order at 7 P.M. by the President, Arthur W. Dean.

About sixty members and guests were present. A buffet supper preceded the meeting.

The President announced the death of the following members of the Society: Ernest W. Danforth, died June 2, 1933, a member since December 21, 1927; Edward F. Miller, died June 12, 1933, a member since November 17, 1897; Louis K. Rourke, died August 23, 1933, a member since November 16, 1910; J. Parker Snow, died September 4, 1933, a member since November 21, 1888; Carroll C. O'Neill, died September 13, 1933, a member since June 17, 1931.

Prof. J. B. Babcock, Secretary, pro tem, announced that the following had been elected to membership:

*Grade of Member:* Joseph A. Power.

*Grade of Junior:* Stephen Haseltine, Jr.

*Grade of Student:* Joseph Dearborn, Anthony Minichiello.

The President announced that Everett N. Hutchins, Secretary of the Society, had been injured recently in an automobile accident. It was *voted* that the sympathy of the Society be extended to Mr. Hutchins, with a hope for a speedy recovery.

The President then introduced the speaker of the evening, Dr. L. Don Leet, seismologist in charge of the Seismograph Station at Oak Ridge Observatory of Harvard University, who gave an illustrated talk on "Earthquakes, Natural and Artificial." After showing views of Harvard's new observatory (which by coincidence is located in the town of Harvard, Mass.) and its equipment, Dr. Leet discussed briefly the nature of earthquakes, the data obtained at seismograph stations, the methods of interpreting the observed results, and some of the geological results of the interpretations.

He concluded his talk with a description of the way in which the principles of interpretation are being applied commercially in seismic methods of prospecting for oil, with dynamite blasts serving as artificial earthquakes.

After an informal discussion and question period the meeting adjourned at 8.30 P.M. with a rising vote of thanks to Dr. Leet for his courtesy in presenting such an interesting paper before the Society.

J. B. BABCOCK,
*Secretary, pro tem.*

### Sanitary Section

JUNE 3, 1933. — Thirty-one members and guests attended the annual excursion of the Section. The first stop was at Shaft 1 on the Newton High-Level Sewer, where the method of chuting concrete into tunnel forms by compressed air was observed. The next stop was made at Shaft 3, where the majority of the party went down into the tunnel and walked into a section which had not reached the stage of lining. The construction of the 60-inch sewer in open cut on Cabot Street was then inspected. The final stop was made at the Newton Water Pumping Station on Needham Street, where extensive alterations and additions were under way.

Frank L. Flood, *Clerk.*

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October 4, 1933. — Supper was enjoyed by 14 members and guests at Patten's new restaurant on Court Street, before the meeting. At 7.15 p.m. the meeting was called to order in the Society rooms, with 23 present. Mr. Stephen DeM. Gage, spoke on his interesting experiences while "Chasing Pollution in Puget Sound." The methods of determining the effect on oyster beds of the discharge into tidal waters, or digester liquors from a large pulp mill, were described. Dr. Wells, of the Harvard School of Public Health, associated with Mr. Gage on this work, contributed a discussion with particular reference to the type of oyster and oyster bed, and his experiments in growing oysters in the waters affected.

Frank L. Flood, *Clerk.*

### Northeastern University Section

October 4, 1933. — Mr. John C. Moses, Chief Engineer of Boston Bridge Works, addressed 35 members of the Section. The subject of his talk dealt with bridge erection problems which he has encountered during his long and distinguished service with the Boston Bridge Works. With the aid of many slides, Mr. Moses showed how it was possible to demolish the old South Station without interference to passengers or train schedules. Accompanying Mr. Moses was Mr. Albert Sanderson, a Northeastern University graduate, who is at present employed at the Boston Bridge Works. Mr. Sanderson described the intricate procedure by which a bridge fabricator obtains a contract.

Professor Henry B. Alvord of Northeastern University announced that a change in the student membership had been made, and that beginning this fall, payment of dues would ensure the student membership for the whole school year.

Bruce Schow, *Clerk.*

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## APPLICATIONS FOR MEMBERSHIP

[October 20, 1933]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

*For Admission*

Dore, Stanley M., Dedham, Mass. (Age 35, b. Somerset, Mass.) Graduate of Brown University, 1920, with degree of B.S. in civil engineering. Experience: June to September, 1919, rodman on survey party on construction work for Scituate water supply reservoir for Providence, R. I.; June, 1920, to February, 1921, draftsman, drawing, tracing and design on reinforced concrete buildings for Turner Construction Company; February, 1921, to November, 1921, engineering draftsman, United States Engineer Office, Providence, R. I., on drafting and design in connection with fortifications; November, 1921, to September, 1928, assistant engineer and assistant designing engineer in designing division of Providence Water Supply Board, on construction of new water supply, including storage reservoir, purification works, pipe lines, aqueducts, control structures, etc.; September, 1928, to date, assistant designing engineer for Metropolitan District Water Supply Commission, on design of structures pertinent to the development of the Swift River and Ware River supplies, including the Quabbin Aqueduct, the Quabbin Dam and Dike, and appurtenant control structures. Refers to *J. F. Brittain, C. L. Coburn, K. R. Kennison, W. H. Sears, F. E. Winsor.*

## ADDITIONS

### *Member*

Francis W. Hamilton, 14 Somerset Road, Brookline, Mass.

### *Students*

Charles M. Kelley, 42 Crest Avenue, Revere, Mass.

Joseph Dearborn, 97 St. Stephen Street, Boston, Mass.

## DEATHS

| | |
|---|---|
| J. Parker Snow . . | Sept. 4, 1933 |
| Carroll C. O'Neill . | Sept. 13, 1933 |

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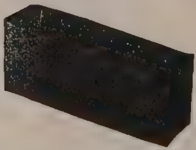

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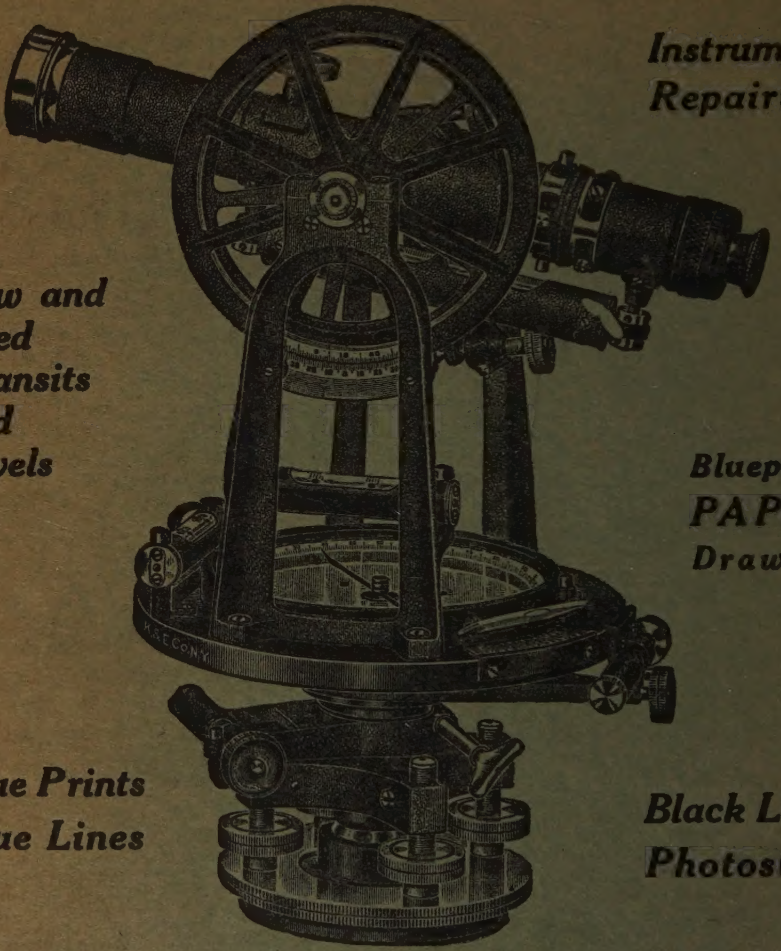

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